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Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Geotechnical Investigation

Proposed Residential Development - Phase 2
114 Richmond Road
Ottawa, Ontario

Prepared For

Ashcroft Homes

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Report: PG2159-1 Revision 4

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa, Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Ashcroft Homes to conduct a geotechnical investigation for Phase 2 of the proposed residential development to be located at 114 Richmond Road, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

- ❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work for this geotechnical investigation.

2.0 Proposed Development

At the present time, it is understood that Phase 2 of the proposed project will consist of four (4) multi-storey residential buildings, each with four (4) to nine (9) storeys above-grade. It is further understood that the proposed buildings will include two (2) levels of underground parking, as well as associated access lanes and landscaped areas.

The northern portion of the subject site is currently occupied by a former convent building, which has been designated as a heritage building. It is understood that the majority of the former convent building will remain in place as part of the proposed development.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechncial investigation was carried out on January 29 and February 1, 2021. During that time, a total of three (3) boreholes were advanced to a maximum depth of 15 m below existing ground surface. The test holes were distributed in a manner to provide general coverage of the proposed development while taking into considered of previous test holes, site features and underground utilities. The test holes completed during the previous geotechnical investigation have been included in this report. The locations of the boreholes are shown on Drawing PG2159-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to our laboratory for further review. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was carried out at six (6) locations (BH 2, BH 3, BH 4, BH 1-21, BH 2-21 and BH 3-21) to confirm the depth to bedrock and to assess its quality. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one drilled section over the length of the drilled section. These values are indicative of the quality of the bedrock.

Subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

A 32 mm PVC groundwater monitoring well was installed in BH 1-21, BH 2-21 and BH 3-21 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details for BH 1-21, BH 2-21, and BH 3-21 are described below:

☐ 3 m of slotted 32 mm diameter PVC screen at the base of borehole.
☐ 32 mm diameter PVC riser pipe from the top of the screen to the ground surface.
☐ No.3 silica sand backfill within annular space around screen.
☐ 300 mm thick bentonite hole plug directly above PVC slotted screen.
☐ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

Hydraulic Conductivity Testing

Hydraulic conductivity testing was completed at three (3) monitoring well locations across the subject site. Falling head tests (“slug tests”) were completed within the glacial till deposit in accordance with ASTM Standard Test Method D4404 - Field Procedure for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers.

Sample Storage

All samples from the current geotechnical investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were determined by Paterson personnel to provide general coverage of the proposed development while taking into consideration previous test holes, site features and underground utilities. The borehole locations and ground surface elevation at each borehole location completed during the current investigation were surveyed by Paterson using a handheld GPS and referenced to geodetic datum

The location and ground surface elevation at each borehole completed during the previous investigation was provided by Annis, O'Sullivan, Vollebekk Ltd..

The borehole location and ground surface elevation at each test hole location are presented on Drawing PG2159-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

Two (2) representative soil samples were submitted for analytical testing to assess the potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The samples was analyzed to determine its concentration of sulphate and chloride along with its resistivity and pH. The laboratory test results are shown in Appendix 1 and the results are discussed in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

Although the majority of the site was snow covered at the time of the field portion of the current geotechnical investigation, it is our understanding that the area was previously used as a staging area for Phase 1 of the proposed development and is known to have been overlain by a layer of granular crushed stone.

The subject section of the site is bordered to the north by an existing heritage building (former convent building) followed by Phase 1 of the current development. Phase 1 of the current development consisted of erecting three (3) adjoining, ten (10) storey structures constructed over three (3) levels of underground parking which are fronting onto Richmond Road. The current phase fo the development is bordered to the south, east and west by mature trees and cedar hedges. The site is bordered to the east by a multi-use pedestrian pathway followed by single family residential dwellings, to the south by Byron Avenue, and to the west by an elementary school and single family residential dwellings. The ground surface generally slopes down in a south to north direction with an approximate difference in ground surface elevations of approximate 2 to 3 m across the subject section of the site.

4.2 Subsurface Profile

The subsurface profile at the borehole locations within the subject section of the site generally consist of topsoil and/or fill extending to a maximum depth of 1.5 m, overlying a glacial till deposit, which in turn is underlain by a grey limestone bedrock. It should be noted that interbedded layers of running sand were encountered within the glacial till deposit at several locations during geotechnical field investigation.

The underlying grey limestone bedrock was cored at BH 2, BH 3, BH 4, BH 1-21, BH 2-21 and BH 3-21 at depths varying between 8.7 and 12.3 m below existing ground surface. Practical refusal to augering was encountered at BH 1 and BH 5 during the previous field investigation at a depth of 8.7 and 9.8 m, respectively on inferred bedrock.

Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Ottawa Formation.

4.3 Groundwater

Groundwater levels (GWLs) recorded at the borehole locations are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
* BH 1-21	70.08	3.03	67.05	February 8, 2021
* BH2-21	68.53	2.12	66.41	February 8, 2021
* BH3-21	68.31	5.70	62.61	February 8, 2021
BH 3	67.85	2.22	65.63	July 20, 2010
BH 4	69.04	1.69	67.35	July 20, 2010
BH 5	68.81	1.02	67.79	July 20, 2010

Note: * denotes boreholes instrumented with monitoring wells.

It should be noted that water levels recorded in the standpipe or groundwater monitoring wells can result in higher than normal groundwater level readings as a result of water that may become trapped within backfilled boreholes. The groundwater level can also be determined based on field observations, such as moisture levels and colouring. Based on these observations, the long-term groundwater level is anticipated to be encountered between an approximate 3 to 4 m depth below existing ground surface.

However, it should be noted that the groundwater is subject to seasonal fluctuations and therefore, the groundwater level could vary at the time of construction.

Hydraulic Conductivity Analysis

Following the completion of the slug testing, the test data was analyzed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous and isotropic aquifer of infinite extent with zero-storage assumption, and a screen length significantly greater than the monitoring well diameter. The assumption regarding aquifer storage is considered to be appropriate for groundwater flow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 3 m and a diameter of 0.051 m.

While the idealized assumptions regarding aquifer extent, homogeneity, and isotropy are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site.

Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale. In cases where the initial hydraulic head displacement is known with relative certainty, such as in this case where a physical slug has been introduced, the line of best fit is considered to pass through the origin.

Based on the above test methods, the monitoring wells screened in the glacial till displayed hydraulic conductivity values ranging from **1.33×10^{-5} to 3.02×10^{-5} m/sec**. The values measured within the monitoring wells are generally consistent with similar material Paterson has encountered on other sites and typical published values for glacial till with a silty sand matrix. These values typically range from 1×10^{-5} to 1×10^{-6} m/sec for a glacial till deposit with a silty sand matrix. The range in hydraulic conductivity values is due to the variability of the material in the deposit. The results of the hydraulic conductivity testing are presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional spread footings placed on the dense glacial till deposit. Alternatively, if design building loads are too high for footings bearing on the dense glacial till deposit to be feasible, consideration should be given to lowering the footings to be supported on the clean, surface sounded bedrock, or to have the footings supported on lean concrete trenches which extend to the clean, surface sounded bedrock.

It is expected that the proposed buildings will constructed lower than the existing heritage building and subsequently within the lateral support zone of the existing footings. As a result, the existing footings may require an underpinning program where the proposed structure is located directly against the existing heritage building. In addition, consideration should also be taken to stabilizing the existing heritage building with hydraulically driven piles extending to the bedrock surface in areas where the temporary shoring system will be located within 6 m of the existing building.

As an alternative to the underpinning discussed above, and in order to lessen impacts to the existing heritage building, it is recommended that the temporary shoring system located within 10 m of the existing heritage building should consist of a concrete secant pile wall due to the bouldery till and the potential for encountering running sand below the long term groundwater table.

The ground surface at the subject site gradually slopes down from the south to the north over an approximate distance of 220 m with an elevation difference of approximately 3 m. Based on the grade differential and property dimensions, a slope of < 2% has been identified across the subject site which has a global stability factor of safety greater than 1.5 and does not require a slope stability study.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under the proposed building and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed buildings, unless otherwise specified, should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building area should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000.

Lean Concrete Filled Trenches

As an alternative to supporting the footings on the dense glacial till deposit, consideration should be given to supporting the footings on vertical, lean concrete trenches (minimum **17 MPa** 28-day compressive strength) which extend to the bedrock surface. Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock surface. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Conventional Spread Footings

Footings founded on an undisturbed, dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **350 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance value at ULS.

An undisturbed bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

Footings designed using the bearing resistance value at SLS provided will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings supported directly on clean surface sounded bedrock, or on lean concrete trenches which extend to the clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5 which was applied to the bearing resistance value at ULS.

A clean, surface sounded bedrock surface bearing surface should be free of all soil and loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations at different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Adequate lateral support is provided to a dense glacial till or engineered fill above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher bearing capacity as the bearing medium soil.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed buildings from Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave interpretation are presented in Appendix 2.

Field Program

The shear wave testing was located towards the southern half of the site, as presented in Drawing PG2159-1 - Test Hole Location Plan presented in Appendix 2. Paterson field personnel placed 24 horizontal geophones in a straight line in roughly a north-south orientation towards the southern side of the site. The 4.5 Hz. horizontal geophones were mounted to the surface by means of a 75 mm ground spike attached to the geophone land case. The geophones were spaced at 3 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between 4 to 8 times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array and at 3, 4.5 and 15 m away from the first and last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building’s foundation.

The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

The glacial till and bedrock velocities were interpreted to be 281 m/s and 1,973 m/s, respectively. It is understood that consideration is being given to founding the buildings directly on bedrock.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012. If the building is founded on glacial till, approximately 3 m above the bedrock surface, the following equation applies:

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = -\frac{30m}{\sum\left(\frac{3m}{281m/s} + \frac{27m}{1973m/s}\right)}$$

$$V_{s30} = 1231m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for foundations placed on the glacial till deposit, and no more than 3 m from the bedrock surface, is 1,231 m/s. Therefore, a **Site Class B** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. Alternatively, if the footings are supported on lean concrete trenches which extend to the bedrock surface, the Vs_{30} would be 1,973 m/s, and a **Site Class A** would apply.

5.5 Basement Slab

It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level are anticipated where a concrete floor slab will be used, it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone.

All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed structures. However, in our opinion, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

It is expected that the foundation will be provided with a perimeter drainage system, therefore, the retained soils should be considered drained. However, if undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight. The total earth pressure (P_{AE}) includes both the static earth pressure component (P_A) and the seismic component (ΔP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_A) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. Note that surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to stay at least 0.3 m away from the walls with the compaction equipment.

Seismic Earth Pressures

The seismic earth pressure (ΔP_{AE}) can be calculated using the earth pressure distribution equal to $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, $9.81\ m/s^2$

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2006. Note that the vertical seismic coefficient is assumed to be zero.

The total earth pressure (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{Pa \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads should be factored as live loads, as per OBC 2006.

5.7 Pavement Structure

Car only parking areas and access lanes are anticipated for the proposed development. The pavement structures presented in Tables 2 and 3 would be applicable for design.

Table 2 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 3 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

Rigid Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below.

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

The building design will incorporate a water suppression system which will consist of a horizontal concrete hydraulic barrier at the base of the excavation and a waterproofing membrane for the vertical surfaces. The water suppression system will reduce water infiltration volumes at post construction which can then be managed by the building sump pit system.

To manage and control groundwater infiltration over the long term, the following water suppression system is recommended to be installed for the foundation walls and underfloor drainage (refer to Figure 4 for an illustration of a typical Groundwater Suppression System in Appendix 2 of this report):

☐ A concrete mud slab will create a horizontal hydraulic barrier to lessen ground water infiltration at the base of the excavation and will consist of a minimum 100 mm thick layer of 25 MPa compressive strength concrete can be considered as an option.

☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at **3 m** below the existing ground surface (which is approximately at the long-term ground water level). The waterproofing membrane will consist of bentonite panels fastened to the temporary shoring system. The membrane should extend to the bottom of the excavation at the founding level of the proposed footings and horizontally over the approved bearing medium and/or concrete mud slab(if chosen) a minimum of 600 mm. Consideration can be given to doubling the bentonite panels in isolated areas where groundwater infiltration is observed to be high at the time of construction.

☐ A composite drainage layer will be placed from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It is expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area

Underfloor Drainage

Underfloor drainage is recommended to control water infiltration below the lowest underground parking level slab that breaches the horizontal hydraulic barrier. For design purposes it is recommended that a 150 mm diameter perforated pipe be placed in each column bay. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available for conventional backfilling, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Elevator Waterproofing System

It is expected that the elevator shaft will extend below the invert level of the underfloor drainage system and will thus be theoretically designed under submerged conditions. As a result, the following elevator shaft waterproofing options should be considered:

Option 1 - Full Waterproofing System

The horizontally applied Colphene BSW H waterproofing membrane (or approved other) should be placed on an adequately prepared mud slab and extend vertically within the inside of the temporary forms of the elevator raft slab. Once the concrete raft slab and elevator shaft sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls. The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the previously applied Colphene BSW H waterproofing membrane installed on the concrete raft slab in accordance with the manufacturers specifications. As a secondary defence, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator pit and the excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 5 - Option 1 - Elevator Waterproofing Detail in Appendix 2 of this report for specific details of the elevator waterproofing.

Option 2 - Partial Waterproofing System

As a result of the size and configuration of the proposed raft slab of the elevator shaft, the following economical waterproofing option can be considered. This option consists of omitting the previously recommended horizontally applied Colphene BSW H waterproofing membrane wrapped around the bottom and sidewalls of the concrete raft slab of the elevator shaft as detailed above in Option 1.

Once the concrete raft slab and elevator pit sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls and horizontally over the elevator raft slab in accordance to the manufacturers specifications. As a secondary defence, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator shaft and the excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 6 - Option 2 - Elevator Waterproofing Detail in Appendix 2 of this report for specific details of the elevator waterproofing

6.3 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

However, the foundations are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.4 Excavation Side Slopes and Temporary Shoring

At this site, temporary shoring will be required to complete the required excavations. However, it is recommended that where sufficient room is available, open cut excavation in combination with temporary shoring can be used.

Excavation Side Slopes

The subsoil at this site is considered to be mainly a Type 2 or Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects. The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below the groundwater level.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

For preliminary design purposes, the temporary system may consist of interlocking sheet piles, secant wall and/or soldier pile and lagging system. However, due to the bouldery glacial till layer, the site may not be suitable for interlocking sheet piling. Furthermore, pre-drilling through the bouldery glacial till may be required as part of the soldier pile and lagging system in advance of driving the soldier piles.

To lessen impacts to the existing heritage building, it is recommended that the temporary shoring system located within 10 m of the existing heritage building should consist of a concrete secant pile wall due to the bouldery till and the potential for encountering running sand below the long term groundwater table.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid damage to adjacent structures and include dewatering control measures. In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Earth pressures acting on the shoring system may be calculated using the parameters provided in Table 5.

Table 5 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Effective (undrained) Unit Weight(γ), kN/m^3	13

Soldier Pile and Lagging System

The earth pressure acting on a soldier pile and lagging shoring system can be calculated using a rectangular earth pressure distribution with a maximum pressure of $0.65 \cdot K \cdot \gamma \cdot H$ for strutted or anchored shoring, or a triangular earth pressure distribution with a maximum value of $K \cdot \gamma \cdot H$ for a cantilever shoring system. H is the height of the excavation.

The active earth pressure should be used where wall movements are permissible while the at-rest pressure should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the undrained unit weights are used for earth pressure calculations, should the level on the groundwater not be lowered below the bottom of the excavation. If the groundwater level is lowered, the total unit weight for the soil should be used for the full height, with no hydrostatic groundwater pressure component.

A minimum factor of safety of 1.5 should be used.

Underpinning Program

As an alternative to the use of a secant pile wall in areas where the proposed building is located in proximity to the existing heritage building, an underpinning program will be required to safely transfer the existing building loads down to a lower founding elevation. Once details fo the proposed building are finalized, it is recommended that details of the underpinning program be prepared in conjunction with the project's structural engineer, if required.

6.5 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.6 Groundwater Control

Although, permeable gravelly sand with silt, cobbles and boulders extend to a depth of approximately 10 m depth across the site, the groundwater level was estimated between 3 to 4 m depth. Based on the falling head tests completed at the borehole locations and water infiltration noted during the first phase of this development, pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) Category 3 permit to take water (PTTW) will be required for this project as it is anticipated that more than 400,000 L/day of groundwater infiltration and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

Impacts on Neighbouring Properties

The overburden at the subject site consists of a topsoil layer overlying a glacial till, which in turn is underlain by limestone bedrock. The majority of the expected groundwater infiltration within the excavations at the subject site will be encountered within the underlying glacial till and bedrock. In regards to the surrounding structures and adjacent roadways, these structures are expected to be founded on glacial till. As such, adverse effects to adjacent structures and nearby roadways related to the short-term duration of the dewatering activities at the subject site are expected to be negligible.

6.7 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.8 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site. The resistivity is indicative of a non aggressive to slightly aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- ☐ Review of the geotechnical aspects of the excavating contractor’s shoring design, prior to construction.
- ☐ Periodic observation of the underpinning program for the adjacent heritage building during the excavation program.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Ashcroft Homes or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

O. Canton

Owen Canton, E.I.T.

Faisal I. Abou-Seido, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Apr. 11, 2022
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

HYDRAULIC CONDUCTIVITY RESULTS

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 13 July 2010

FILE NO. PG2159

HOLE NO. BH 1

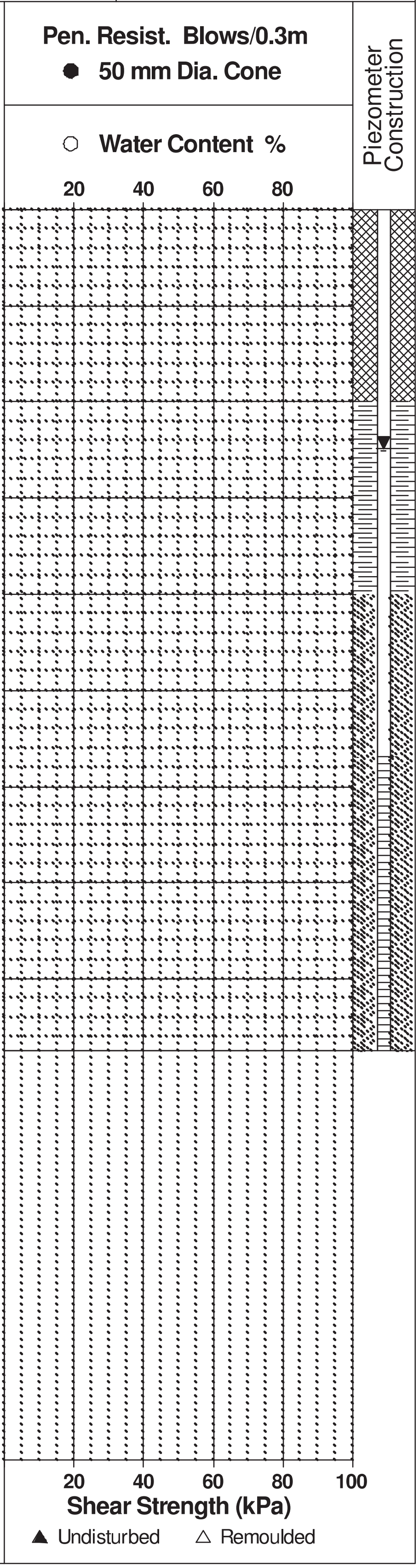
SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL 0.13		AU	1		
GLACIAL TILL: Dense to very dense, brown gravelly sand with silt, cobbles and boulders		SS	2		34
		SS	3	58	82
2.90		SS	4	56	50+
GLACIAL TILL: Loose to compact, grey silty sandy clay with gravel, cobbles and boulders		SS	5	100	1
		SS	6	42	2
		SS	7	83	2
		SS	8	100	1
		SS	9	33	16
8.74		SS	10	75	16

End of Borehole

Practical refusal to augering @ 8.74m depth

(GWL @ 2.48m-July 20/10)

DEPTH (m)	ELEV. (m)
0	68.42
1	67.42
2	66.42
3	65.42
4	64.42
5	63.42
6	62.42
7	61.42
8	60.42



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

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REMARKS

BORINGS BY CME 55 Power Auger

DATE 13 July 2010

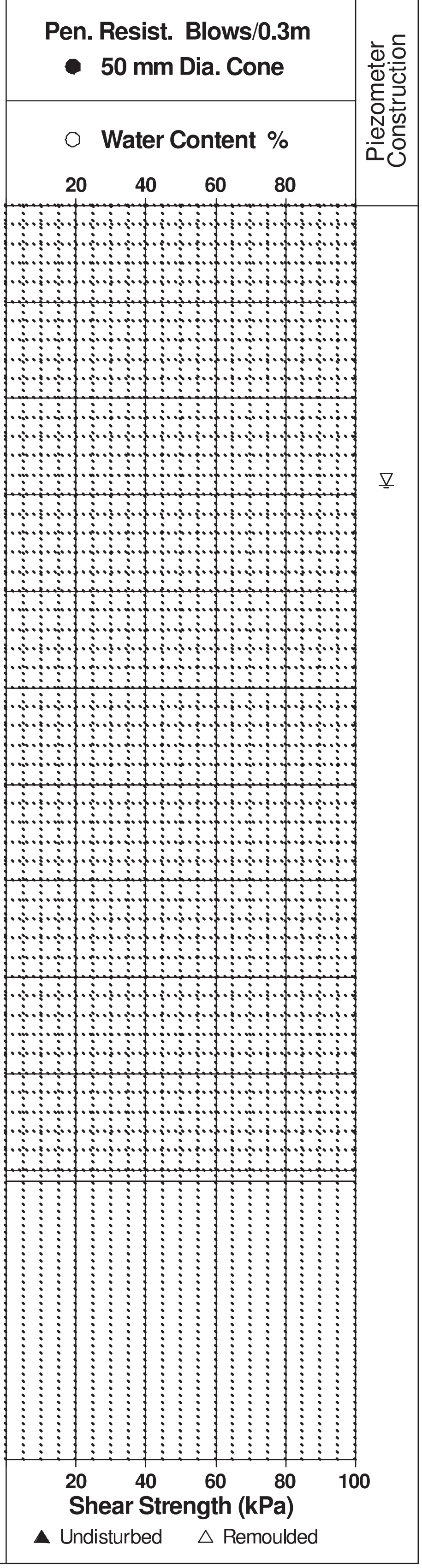
FILE NO. PG2159

HOLE NO. BH 2

SOIL DESCRIPTION	
GROUND SURFACE	
TOPSOIL	0.10
GLACIAL TILL: Very dense, brown gravelly sand with silt, cobbles and boulders	3.66
GLACIAL TILL: Loose, grey silty sandy clay with gravel and cobbles	7.00
GLACIAL TILL: Dense, grey silty sand with gravel and cobbles	9.02
BEDROCK: Grey limestone	10.11
End of Borehole (GWL @ 2.9m depth based on field observations)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	91
SS	3	91	50+
SS	4	100	50+
SS	5	80	50+
SS	6	50	6
SS	7	67	4
SS	8	62	2
SS	9	38	9
SS	10	50	32
RC	1	100	80

DEPTH (m)	ELEV. (m)
0	68.31
1	67.31
2	66.31
3	65.31
4	64.31
5	63.31
6	62.31
7	61.31
8	60.31
9	59.31
10	58.31



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG2159

REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger

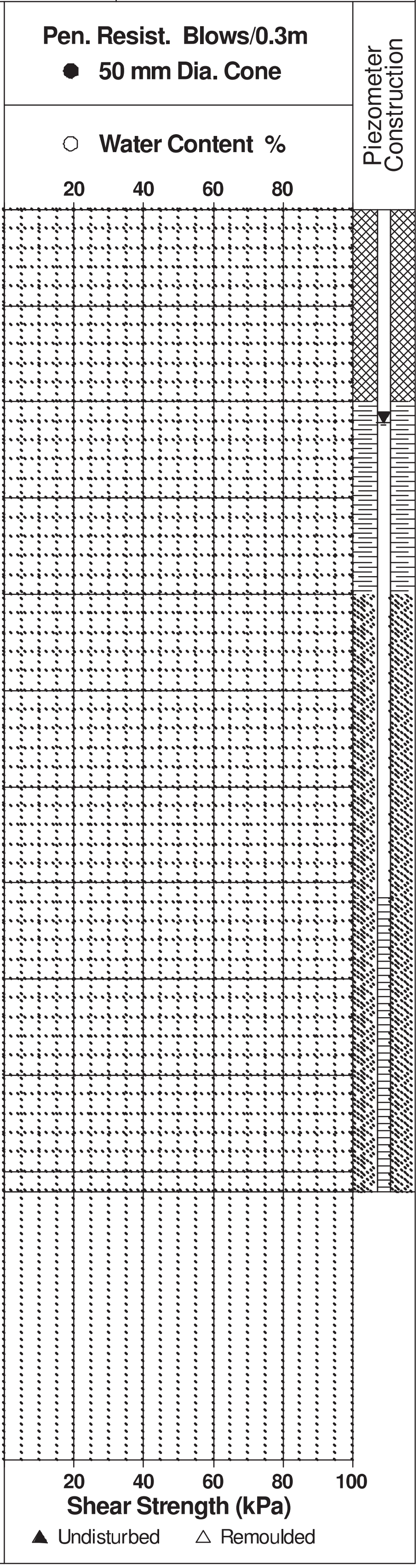
DATE 13 July 2010

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.30	AU	1		
GLACIAL TILL: Loose, grey silty clay with sand and gravel	1.45	SS	2	29	2
GLACIAL TILL: Compact to very dense, grey silty sand with gravel, cobbles and boulders	8.69	SS	3	100	
		SS	4	100	26
		SS	5	50	
		SS	6	100	50+
		SS	7	100	12
		SS	8	100	50+
		SS	9	100	50+
		SS	10	91	50+
		SS	11	79	53
BEDROCK: Grey limestone	10.21	RC	1	100	29

End of Borehole

(GWL @ 2.22m-July 20/10)

DEPTH (m)	ELEV. (m)
0	67.85
1	66.85
2	65.85
3	64.85
4	63.85
5	62.85
6	61.85
7	60.85
8	59.85
9	58.85
10	57.85



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SOIL PROFILE AND TEST DATA

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Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

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REMARKS

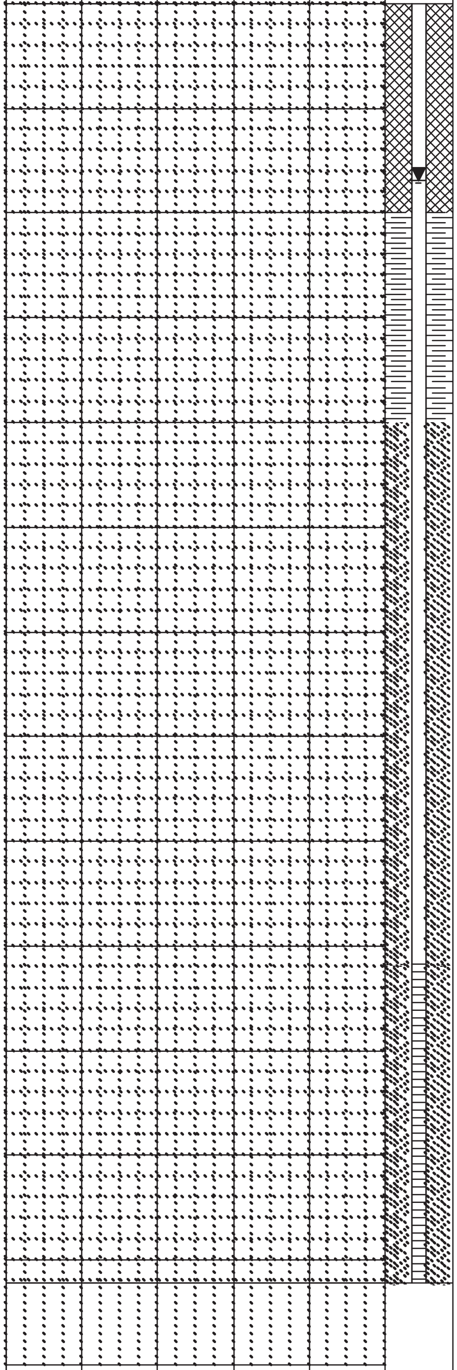
BORINGS BY CME 55 Power Auger

DATE 13 July 2010

FILE NO. PG2159

HOLE NO. BH 4

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
TOPSOIL	0.30	AU	1			0	69.04
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, trace clay - grey by 3.7m depth		SS	2		43	1	68.04
		SS	3	75	52	2	67.04
						3	66.04
		SS	4	100	64		
		SS	5	100	50+	4	65.04
		SS	6		50+	5	64.04
						6	63.04
		SS	7	100	50+		
		SS	8	100	58	7	62.04
		SS	9		73	8	61.04
						9	60.04
	10.69					10	59.04
BEDROCK: Grey limestone						11	58.04
		RC	1	100	21		
	12.22					12	57.04
End of Borehole (GWL @ 1.69m-July 20/10)							



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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 13 July 2010

FILE NO. PG2159

HOLE NO. BH 5

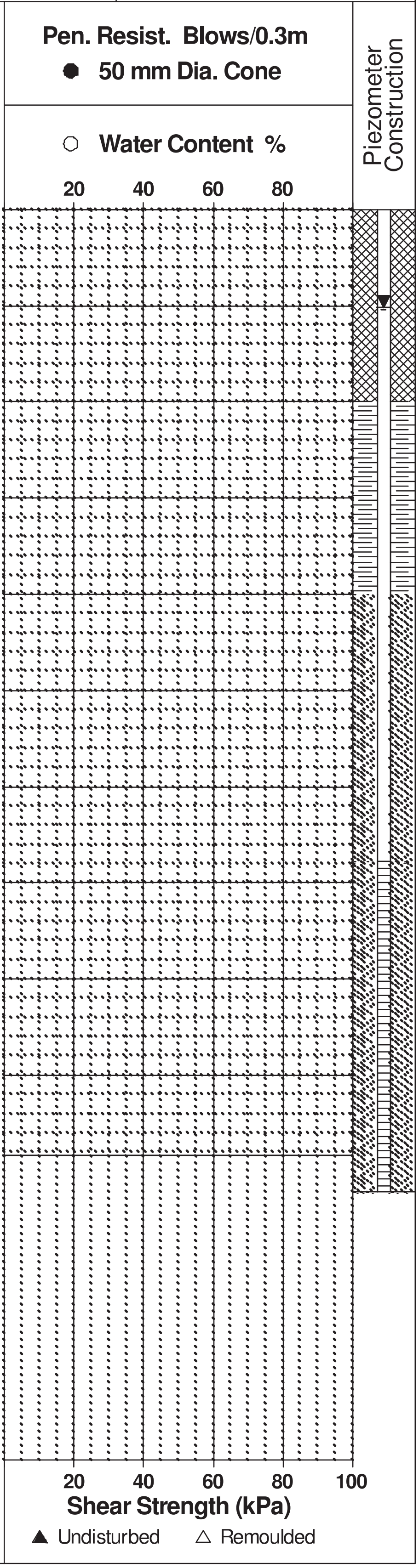
SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE						
TOPSOIL	0.30	■	AU	1		
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, trace clay - grey by 3.0m depth		^^^	SS	2	67	17
			SS	3	78	50+
			SS	4		37
			SS	5	75	74
			SS	6	100	47
			SS	7	100	50+
			SS	8		51
			SS	9	100	50+
	9.83					

End of Borehole

Practical refusal to augering @ 9.83m depth

(GWL @ 1.02m-July 20/10)

DEPTH (m)	ELEV. (m)
0	68.81
1	67.81
2	66.81
3	65.81
4	64.81
5	63.81
6	62.81
7	61.81
8	60.81
9	59.81



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG2159

REMARKS

HOLE NO. BH 1-21

BORINGS BY CME-55 Low Clearance Drill **DATE** 2021 February 1

SOIL DESCRIPTION	Strata bottom (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty sand with gravel, crushed stone, some cobbles, trace organics	0.71	AU	1		
Stiff brown **SILTY CLAY** with some sand, trace organics	1.93	SS	2	67	14
		SS	3	88	72
GLACIAL TILL: Grey silty sand with gravel, some clay, trace cobbles occasional boulders	3.66	SS	4	79	+50
		SS	5	0	+50
GLACIAL TILL: Dense brown silty sand with gravel, some cobbles - Some course sand by 4.5 m depth - Some running sand encountered between 4.5 m and 7.6 m	7.62	SS	6	63	+50
		SS	7	54	+50
		SS	8	0	+50
		SS	9	75	+50
GLACIAL TILL: Compact grey silty sand, some gravel, cobbles and occasional boulders	12.27	SS	10	54	16
		RC	1	100	60
BEDROCK: Good to excellent quality grey Limestone	14.99	RC	2	100	100
End of Borehole (GWL @ 3.86 m depth - Feb 8, 2021)					

DEPTH (m)	ELEV. (m)
0	70.08
1	69.08
2	68.08
3	67.08
4	66.08
5	65.08
6	64.08
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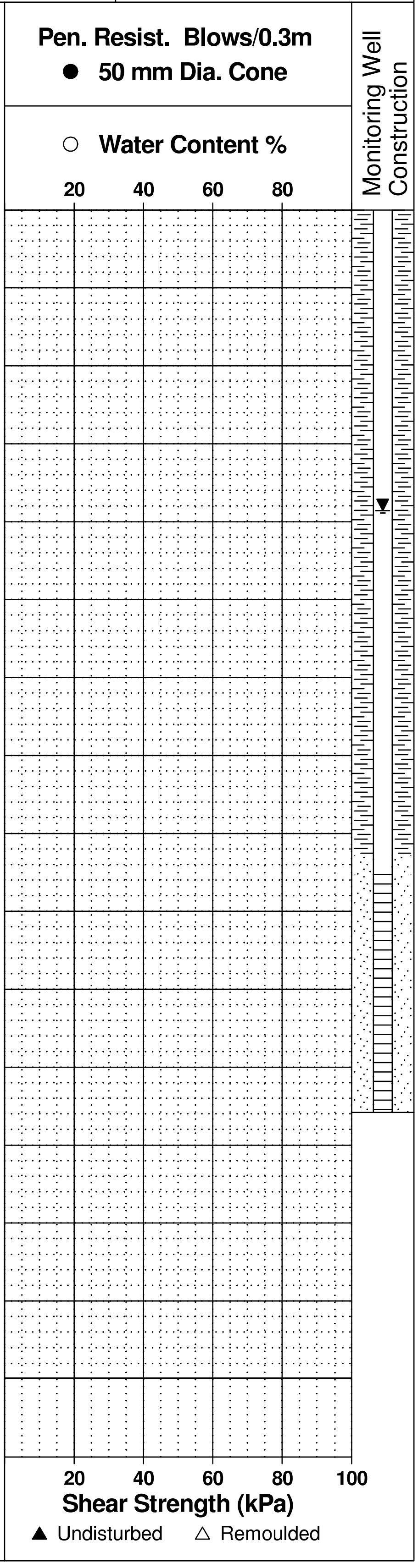
9	61.08
10	60.08
11	59.08
12	58.08
13	57.08
14	56.08



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

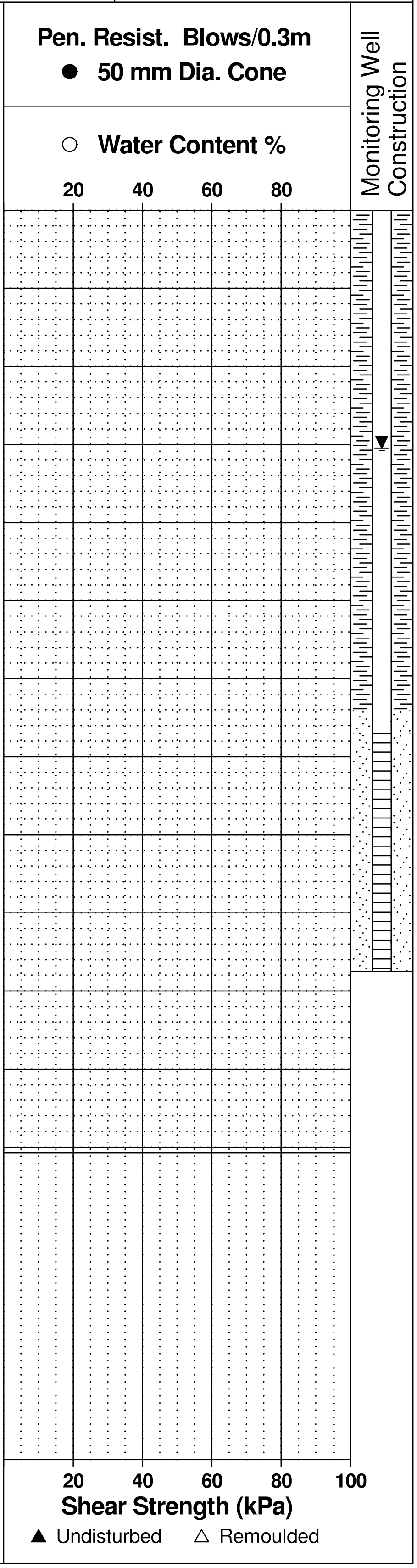
BORINGS BY CME-55 Low Clearance Drill

DATE 2021 February 1

FILE NO. PG2159

HOLE NO. BH 2-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	68.53
FILL: Brown silty sand with crushed stone trace organics and topsoil	AU	1				
	SS	2	83	7	1	67.53
1.35						
TOPSOIL some gravel						
1.52						
GLACIAL TILL: Compact to dense, grey silty sand with clay, gravel, trace cobbles - Decreasing clay content with depth	SS	3	58	13	2	66.53
	SS	4	0	+50	3	65.53
	SS	5	67	65		
	SS	6	63	51	4	64.53
4.57						
GLACIAL TILL: Grey silty sand trace gravel - Some running sand encountered from 4.5 m to 6.1 m depth - Some gravel, cobbles and occasional boulders by 6.1 m depth	SS	7	4	29	5	63.53
	SS	8	92	28	6	62.53
	SS	9	21	36	7	61.53
					8	60.53
					9	59.53
9.83						
BEDROCK: Poor to good quality grey Limestone	RC	1	100	39	10	58.53
	RC	2	100	69	11	57.53
12.07					12	56.53
End of Borehole (GWL @ 3.05 m depth - Feb 8, 2021)						



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development-114 Richmond Rd.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 February 1

FILE NO. PG2159

HOLE NO. BH 3-21

GROUND SURFACE

SOIL DESCRIPTION	Depth (m)
FILL: Brown silty sand with crushed stone	0.30
	0.66
FILL: Brown silty sand trace clay and gravel	1.52
FILL: Brown silty sand some clay	1.73
TOPSOIL	2.13
Brown **CLAYEY SILT** with organics	
Stiff brown **SILTY CLAY**	2.97
GLACIAL TILL brown silty sand, trace gravel - Occasional cobbles by 5.4 m depth	7.62
GLACIAL TILL: Brown silty sand to sandy silt with gravel, cobbles and occasional boulders	9.75
BEDROCK: Fair to good quality grey Limestone	12.67

End of Borehole

(GWL @ 6.54 m depth - Feb 9, 2021)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	42	10
SS	3	46	3
SS	4	100	2
SS	5	63	37
SS	6	100	56
SS	7	79	63
SS	8	17	+50
SS	9	38	+50
SS	10	33	+50
RC	1	100	53
RC	2	97	76

DEPTH (m)	ELEV. (m)
0	68.31
1	67.31
2	66.31
3	65.31
4	64.31
5	63.31
6	62.31
7	61.31
8	60.31
9	59.31
10	58.31
11	57.31
12	56.31

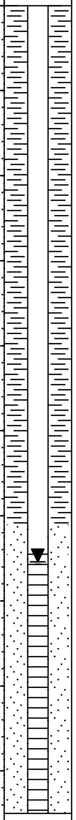
Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**

○ **Water Content %**

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Monitoring Well Construction



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

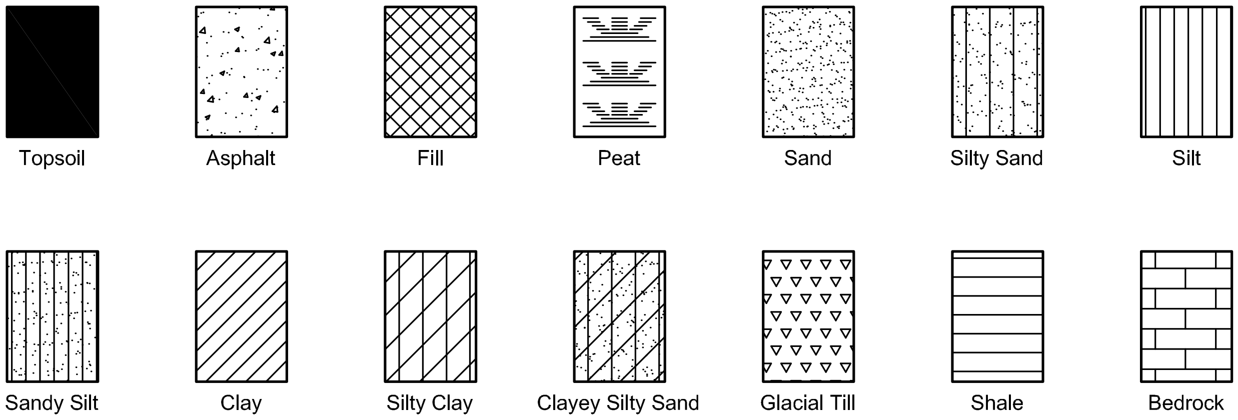
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

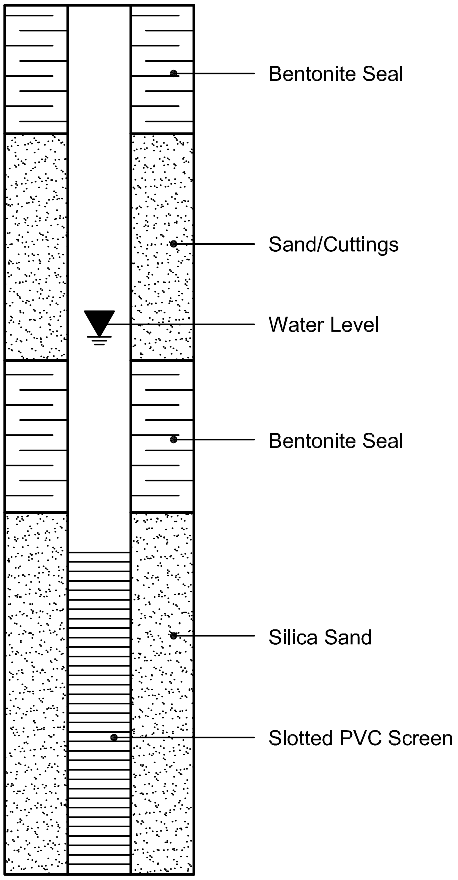
SYMBOLS AND TERMS (continued)

STRATA PLOT

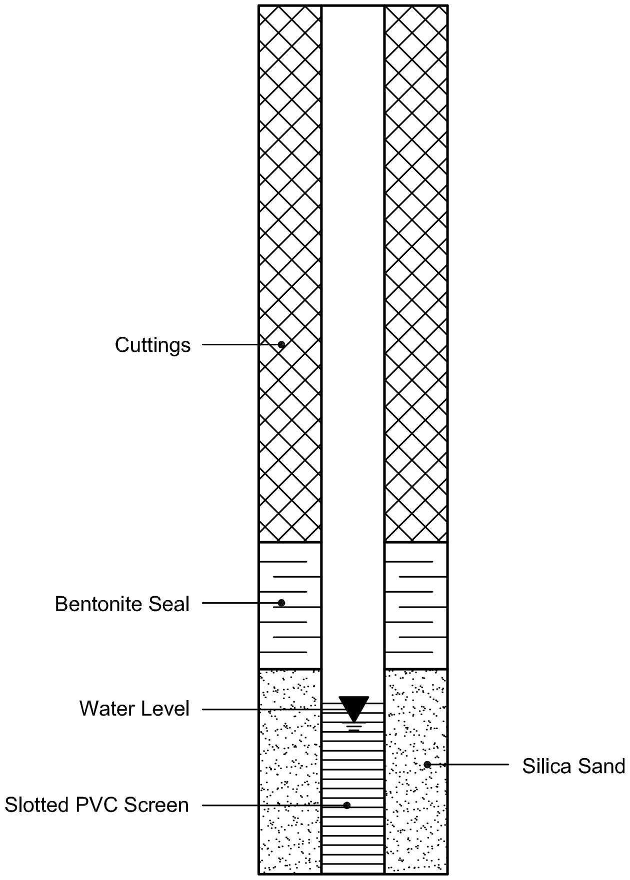


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Report Date: 21-Jul-2010
Order Date:19-Jul-2010

Client: **Paterson Group Consulting Engineers**
Client PO: 8997
Project Description: PG2159

	Client ID:	BH3 SS8	-	-	-
	Sample Date:	14-Jul-10	-	-	-
	Sample ID:	1030028-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	82.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.93	-	-	-
Resistivity	0.10 Ohm.m	59.8	-	-	-
Anions					
Chloride	5 ug/g dry	32	-	-	-
Sulphate	5 ug/g dry	36	-	-	-

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31891

Report Date: 08-Feb-2021

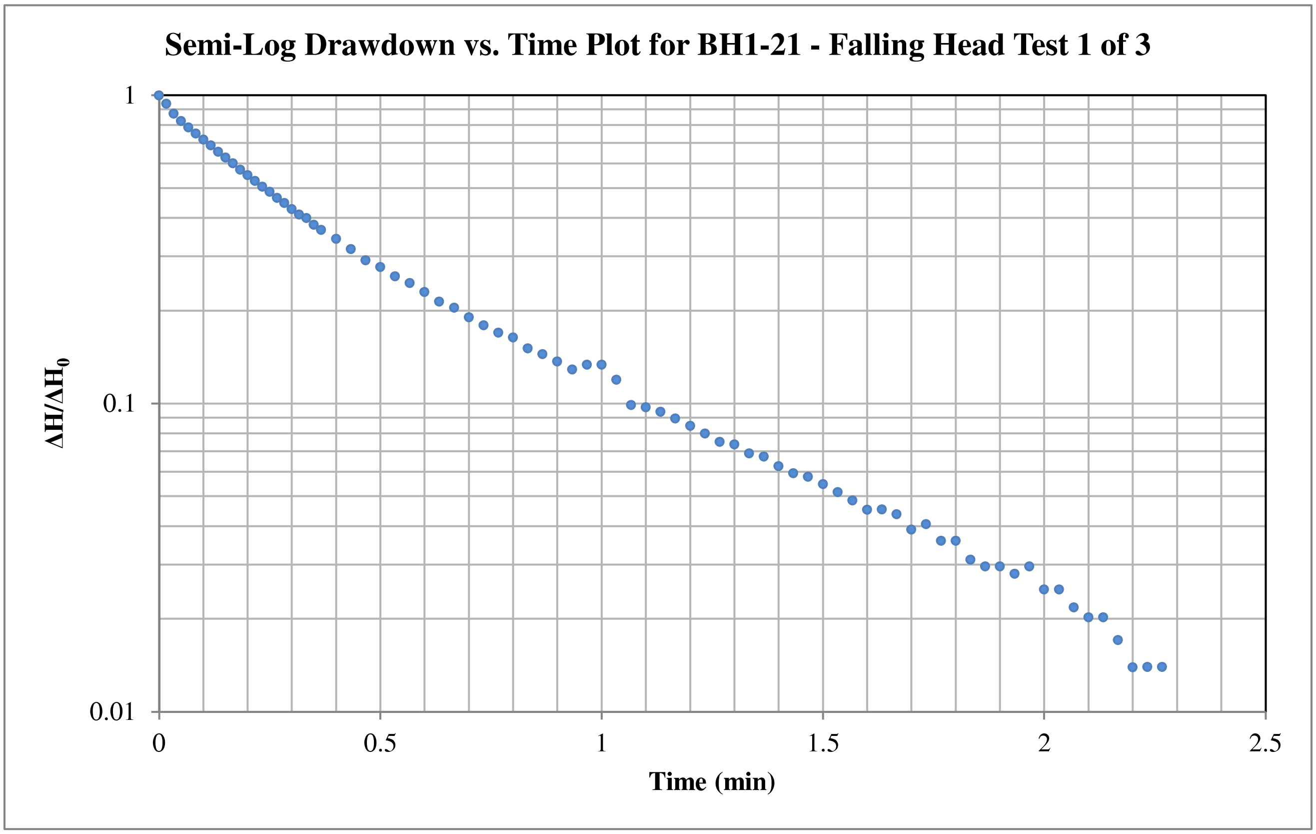
Order Date: 2-Feb-2021

Project Description: PG2159

	Client ID:	BH1-21-SS4	-	-	-
	Sample Date:	29-Jan-21 13:00	-	-	-
	Sample ID:	2106205-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	91.5	-	-	-
General Inorganics					
pH	0.05 pH Units	7.39	-	-	-
Resistivity	0.10 Ohm.m	65.1	-	-	-
Anions					
Chloride	5 ug/g dry	15	-	-	-
Sulphate	5 ug/g dry	85	-	-	-

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH1-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

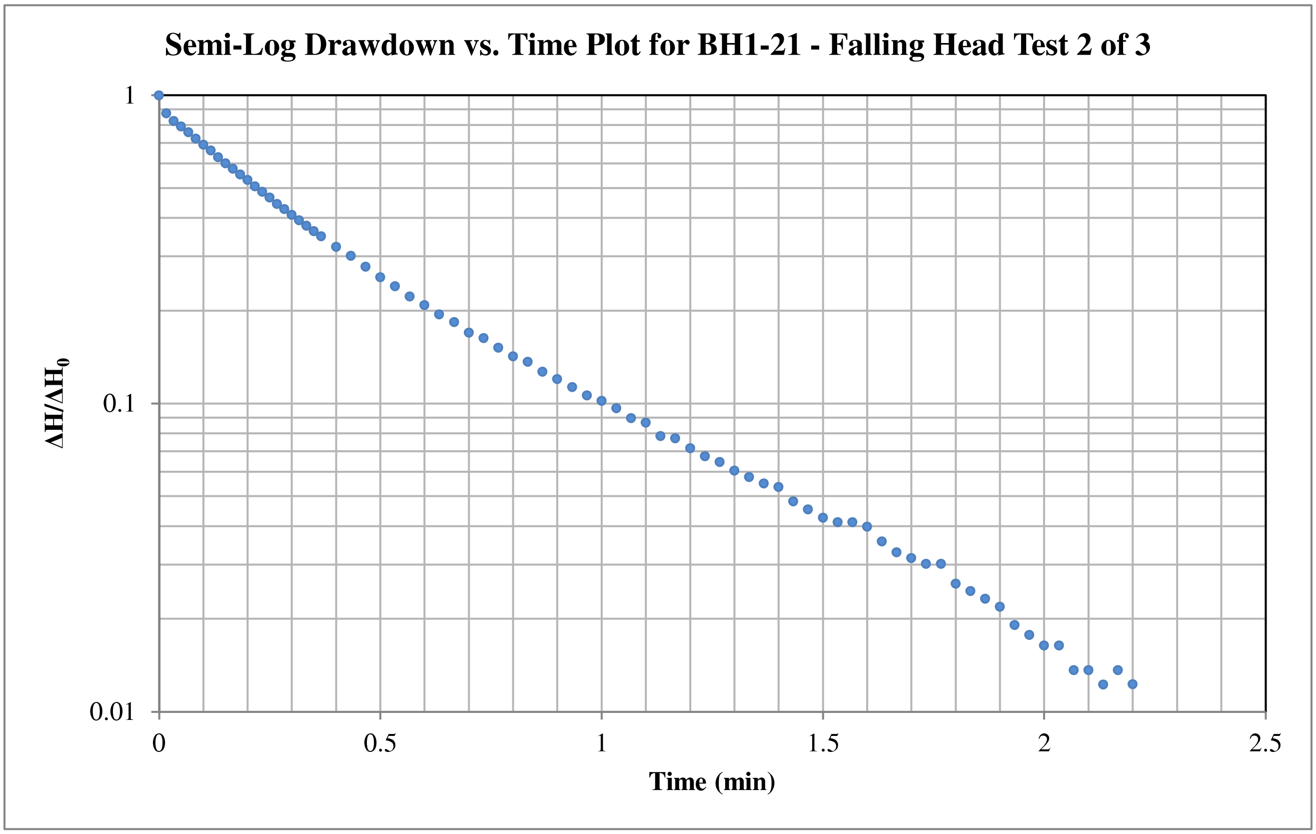
Data Points (from plot):

t*: 0.363 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.36E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH1-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^{2}}{F}\frac{1}{t*}\ln\left(\frac{\Delta H *}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

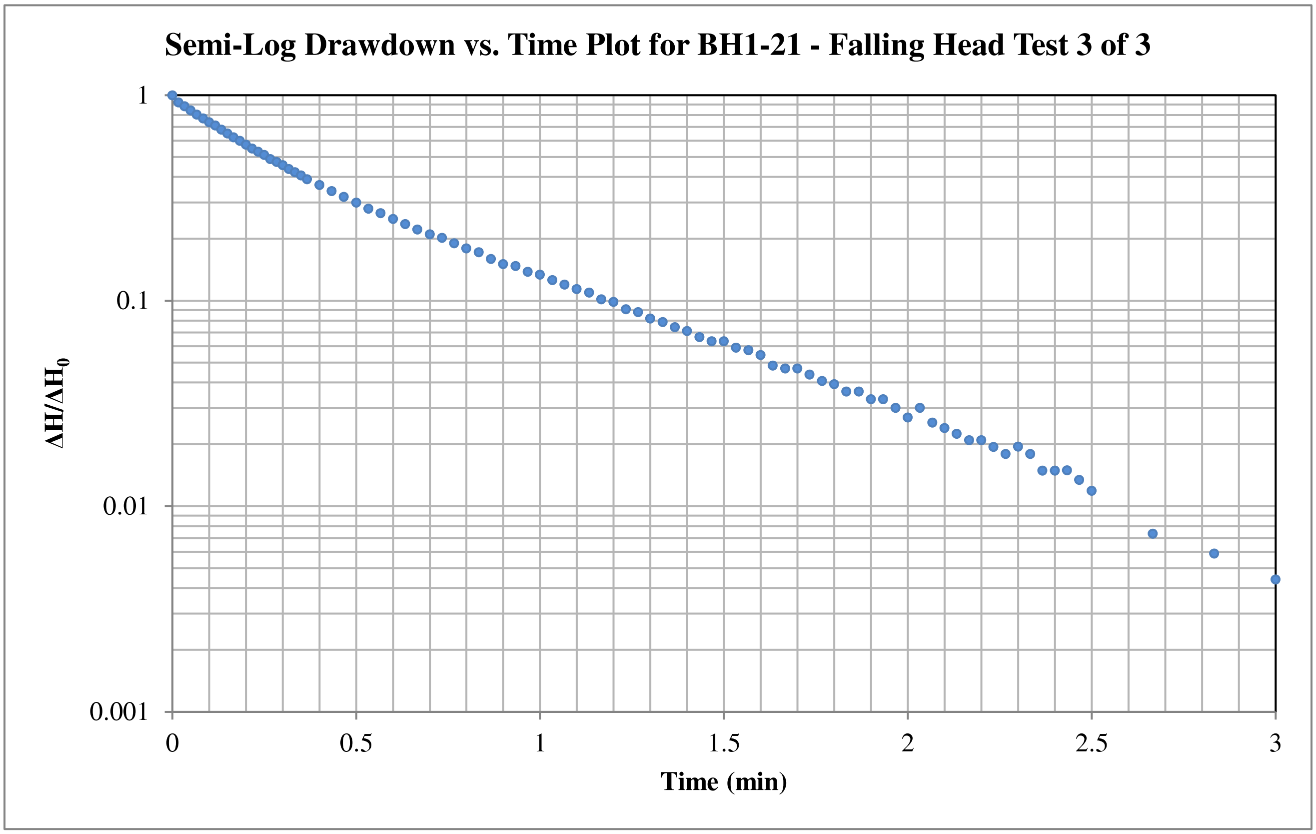
Data Points (from plot):

t*: 0.341 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.51E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH1-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

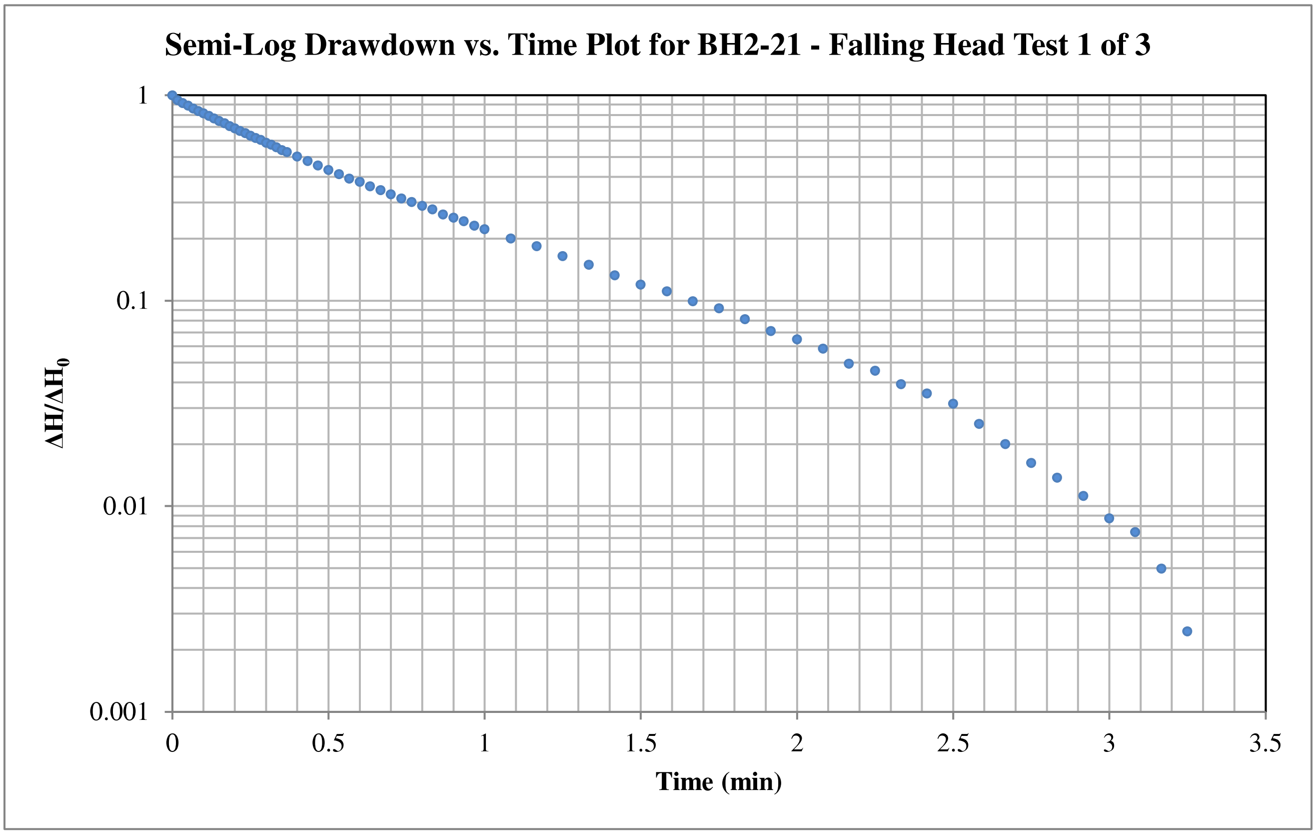
Data Points (from plot):

t*: 0.395 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.17E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH2-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

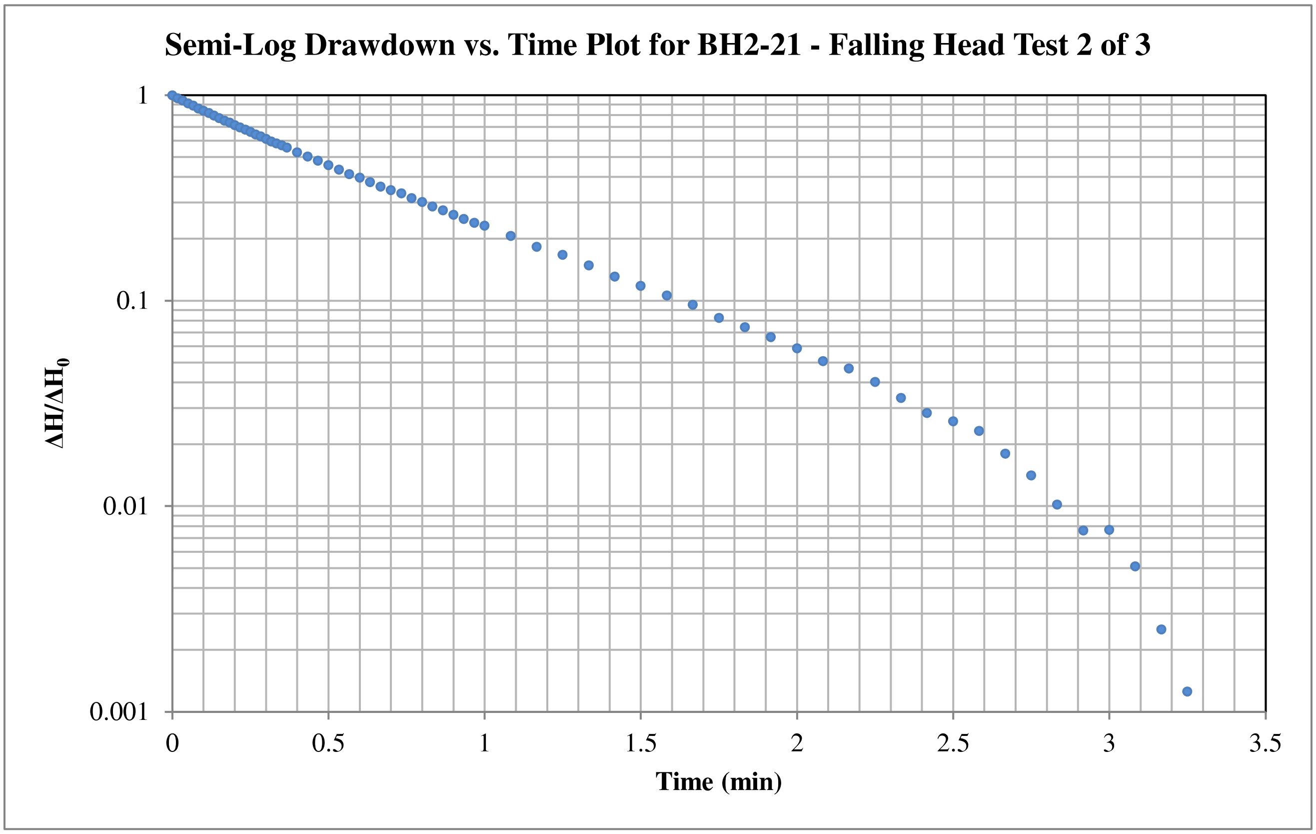
Data Points (from plot):

t^*: 0.615 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.39E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH2-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^2}{F}\frac{1}{t^*}\ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3 m	Saturated length of screen or open hole
D	0.051 m	Diameter of well
r_c	0.0255 m	Radius of well

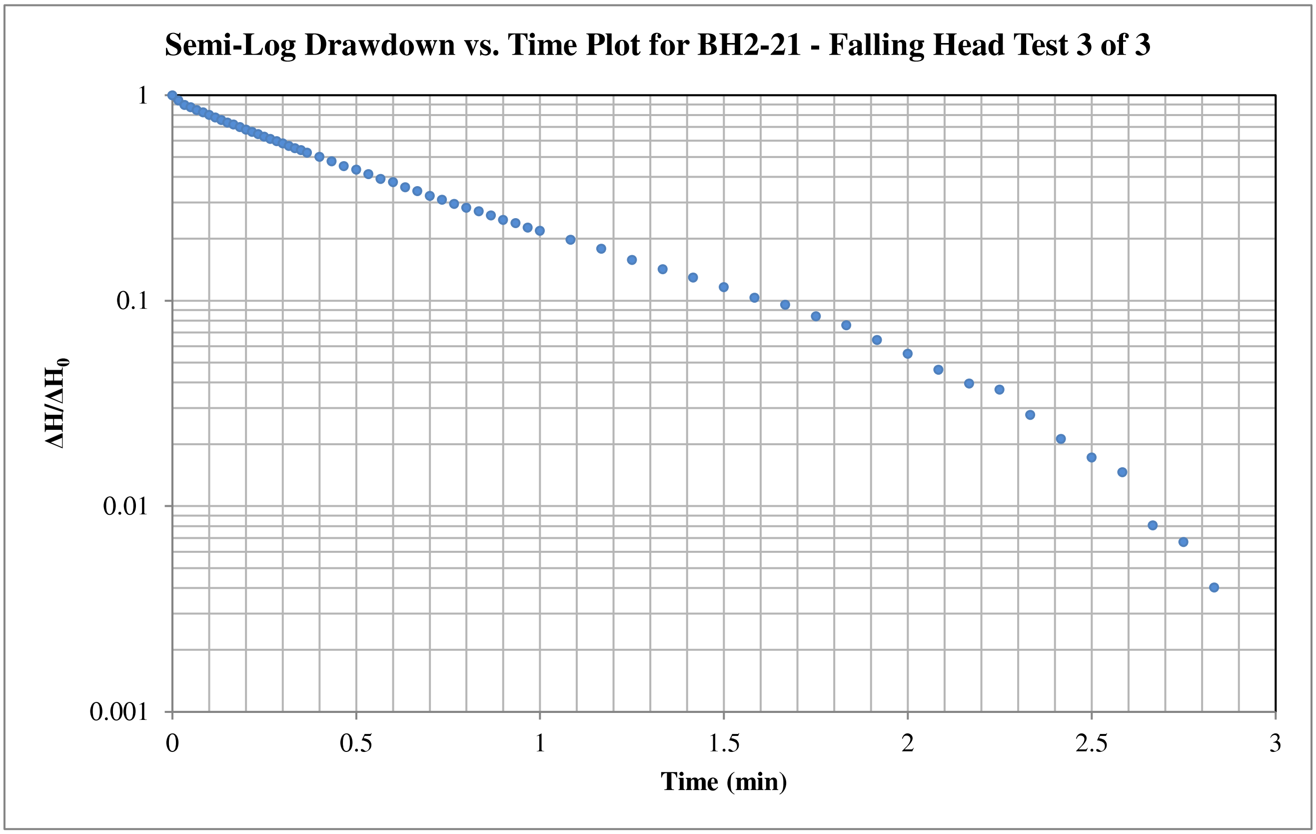
Data Points (from plot):

t*: 0.646 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.33E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH2-21
Test: Falling Head
Date: February 8, 2021


Semi-Log Drawdown vs. Time Plot for BH2-21 - Falling Head Test 3 of 3

Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^{\,2}}{F}\frac{1}{t*}\ln\left(\frac{\Delta H\ *}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

Data Points (from plot):

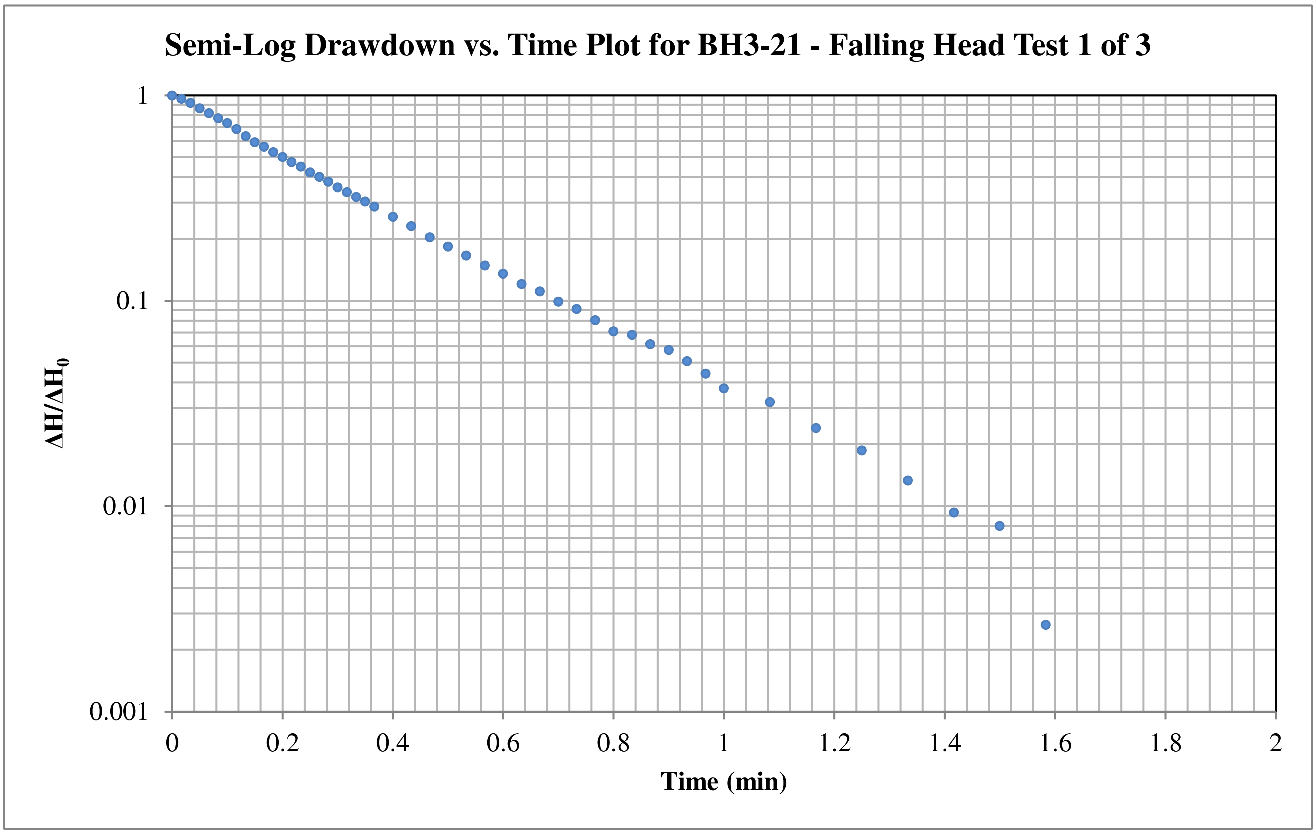
t*: 0.608 minutes

$\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 1.41E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH3-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$\boxed{K = \frac{\pi r_c^{2}}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)}$$

Hvorslev Shape Factor

$$\boxed{F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

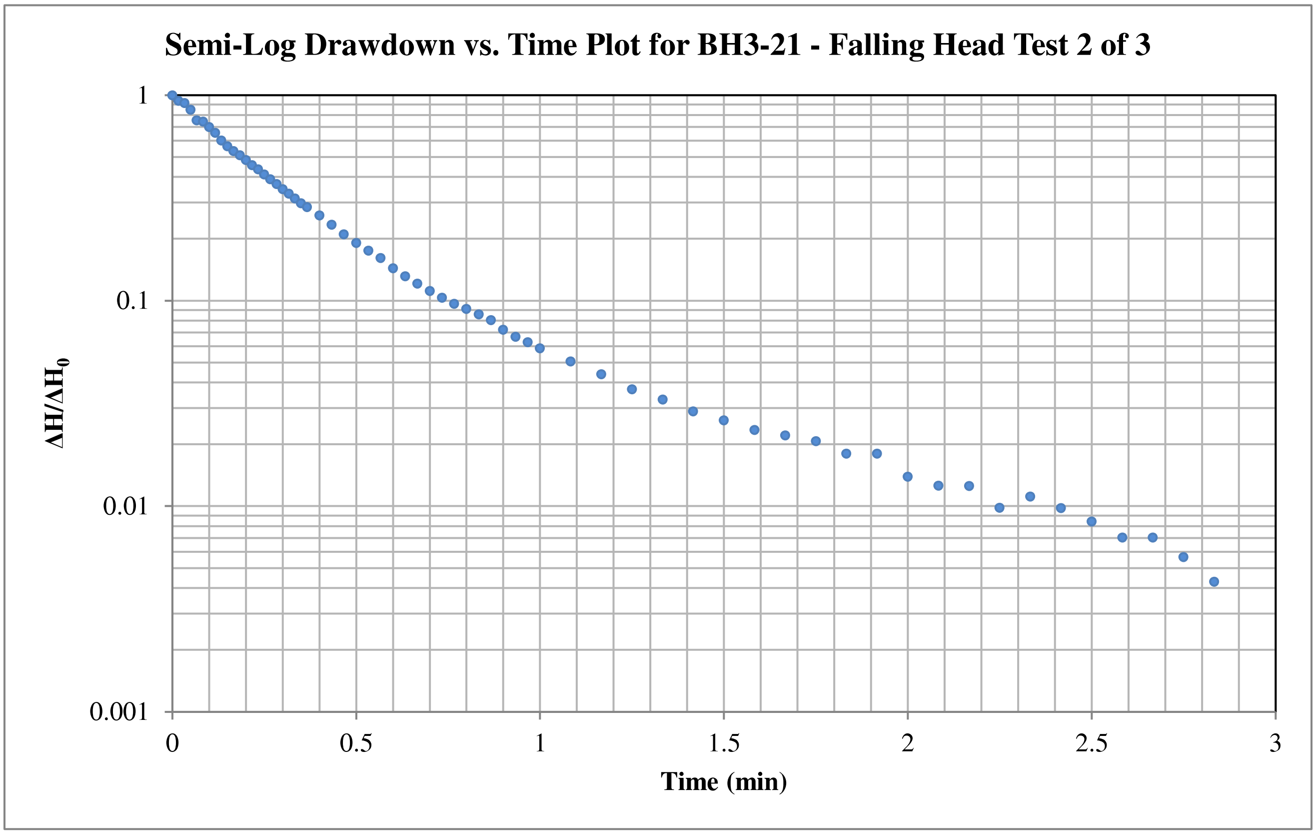
Data Points (from plot):

t*: 0.291 minutes ΔH*/ΔH$_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.94E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH3-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

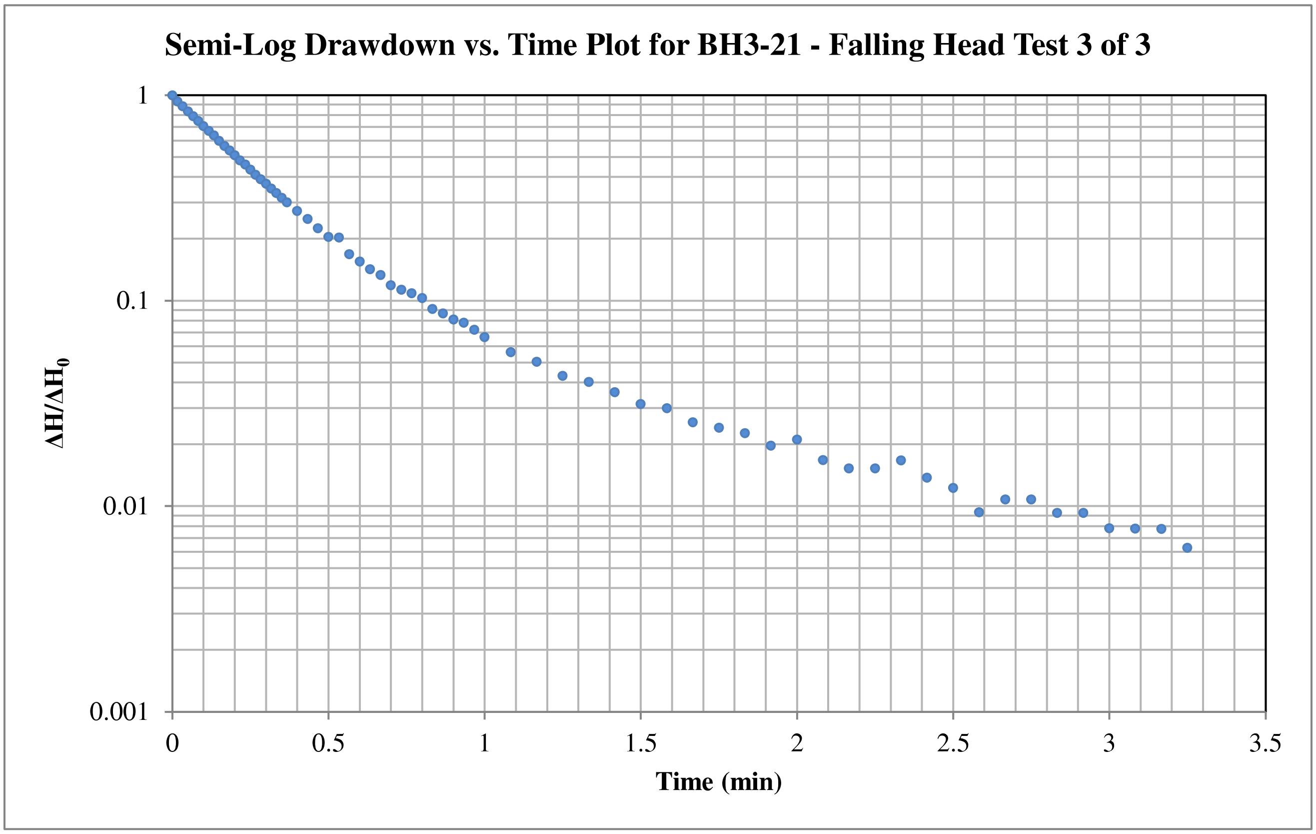
Data Points (from plot):

t*: 0.283 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 3.02E-05 m/sec

Hvorslev Hydraulic Conductivity Analysis

Project: Ashcroft - 114 Richmond Road
Test Location: BH3-21
Test: Falling Head
Date: February 8, 2021



Hvorslev Horizontal Hydraulic Conductivity

$$K = \frac{\pi r_c^2}{F} \frac{1}{t^*} \ln\left(\frac{\Delta H^*}{\Delta H_0}\right)$$

Hvorslev Shape Factor

$$F = \frac{2\pi L}{\ln\left(\frac{2L}{D}\right)}$$

Valid for L>>D

Hvorslev Shape Factor F: 3.9536

Well Parameters:

L	3	m	Saturated length of screen or open hole
D	0.051	m	Diameter of well
r_c	0.0255	m	Radius of well

Data Points (from plot):

t*: 0.301 minutes $\Delta H^*/\Delta H_0$: 0.37

Horizontal Hydraulic Conductivity
K = 2.85E-05 m/sec

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

FIGURE 4 - GROUNDWATER SUPPRESSION SYSTEM

FIGURE 5 - OPTION 1 - ELEVATOR WATERPROOFING DETAIL

FIGURE 6 - OPTION 2 - ELEVATOR WATERPROOFING DETAIL

DRAWING PG2159-1 - TEST HOLE LOCATION PLAN

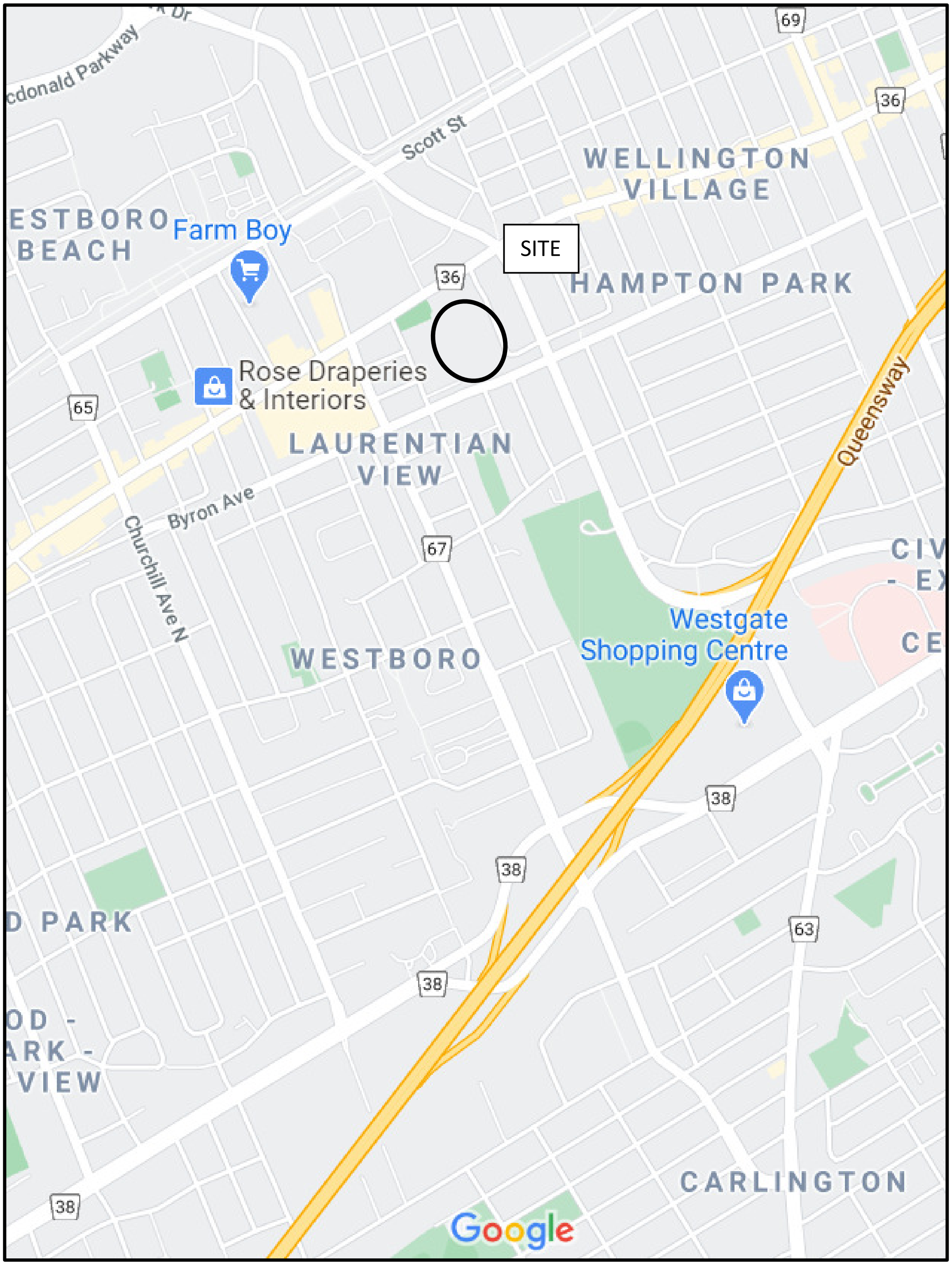


FIGURE 1

KEY PLAN

patersongroup

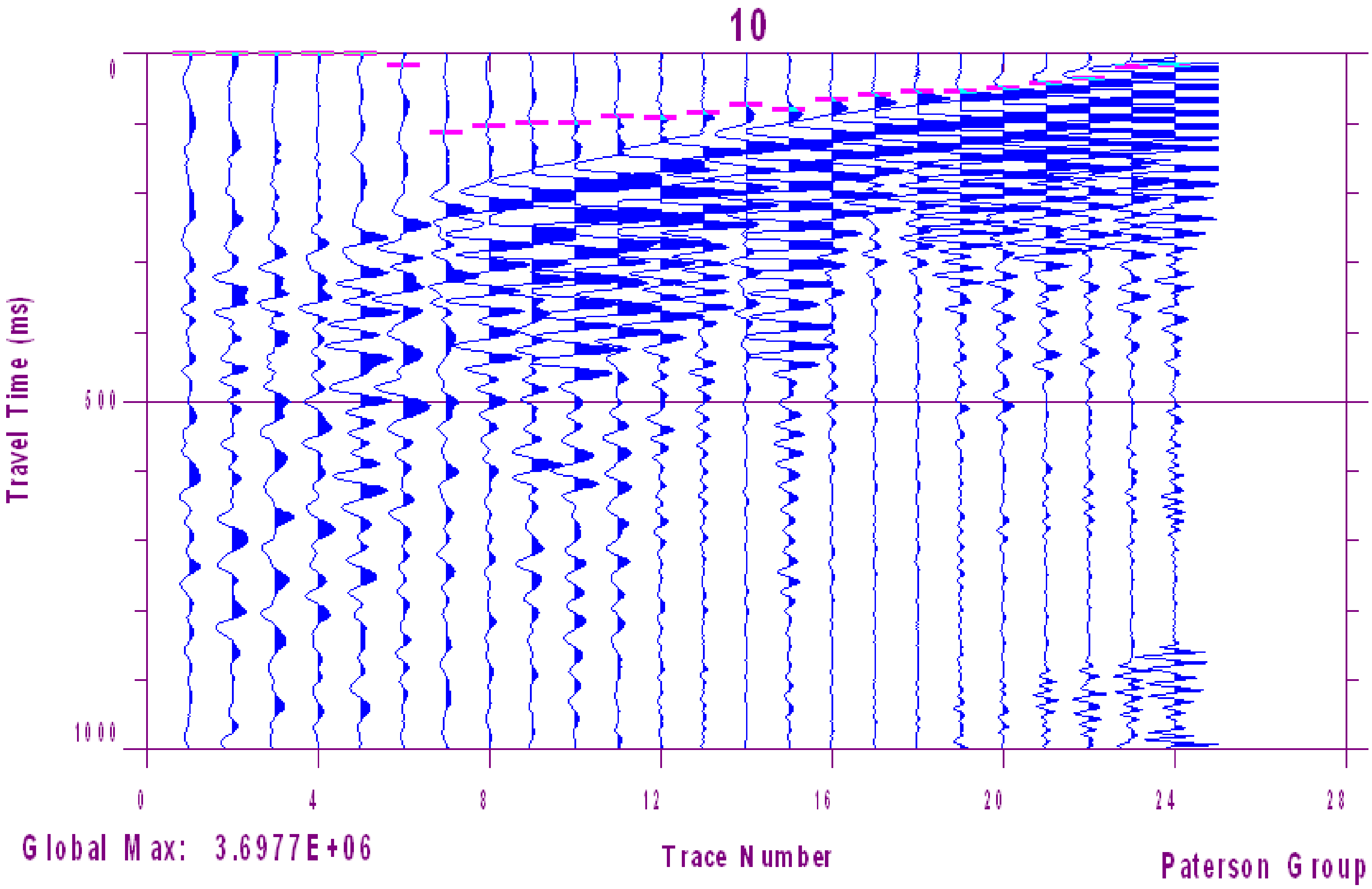


Figure 2 – Shear Wave Profile at Shot Location 72.0 m

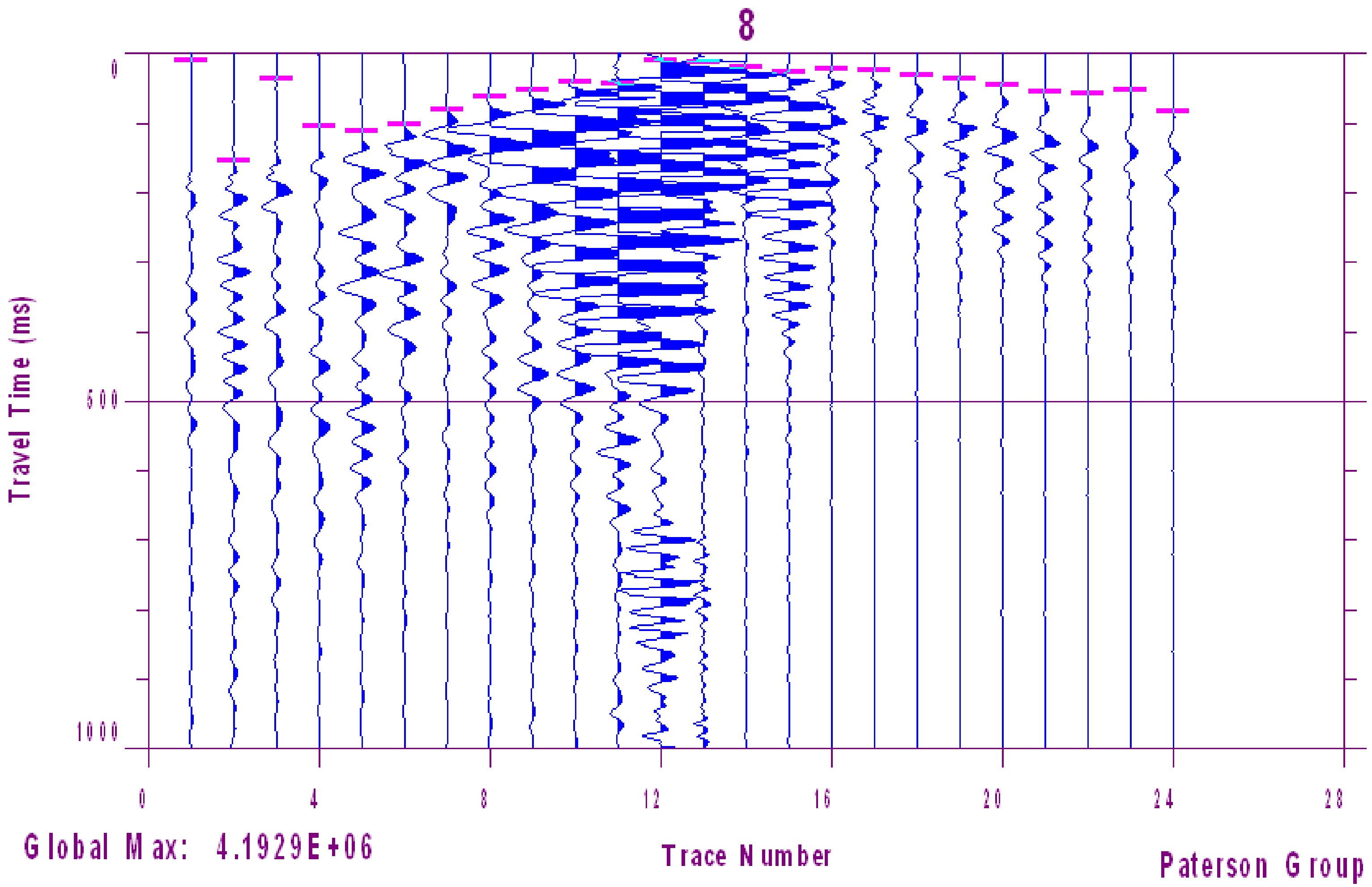
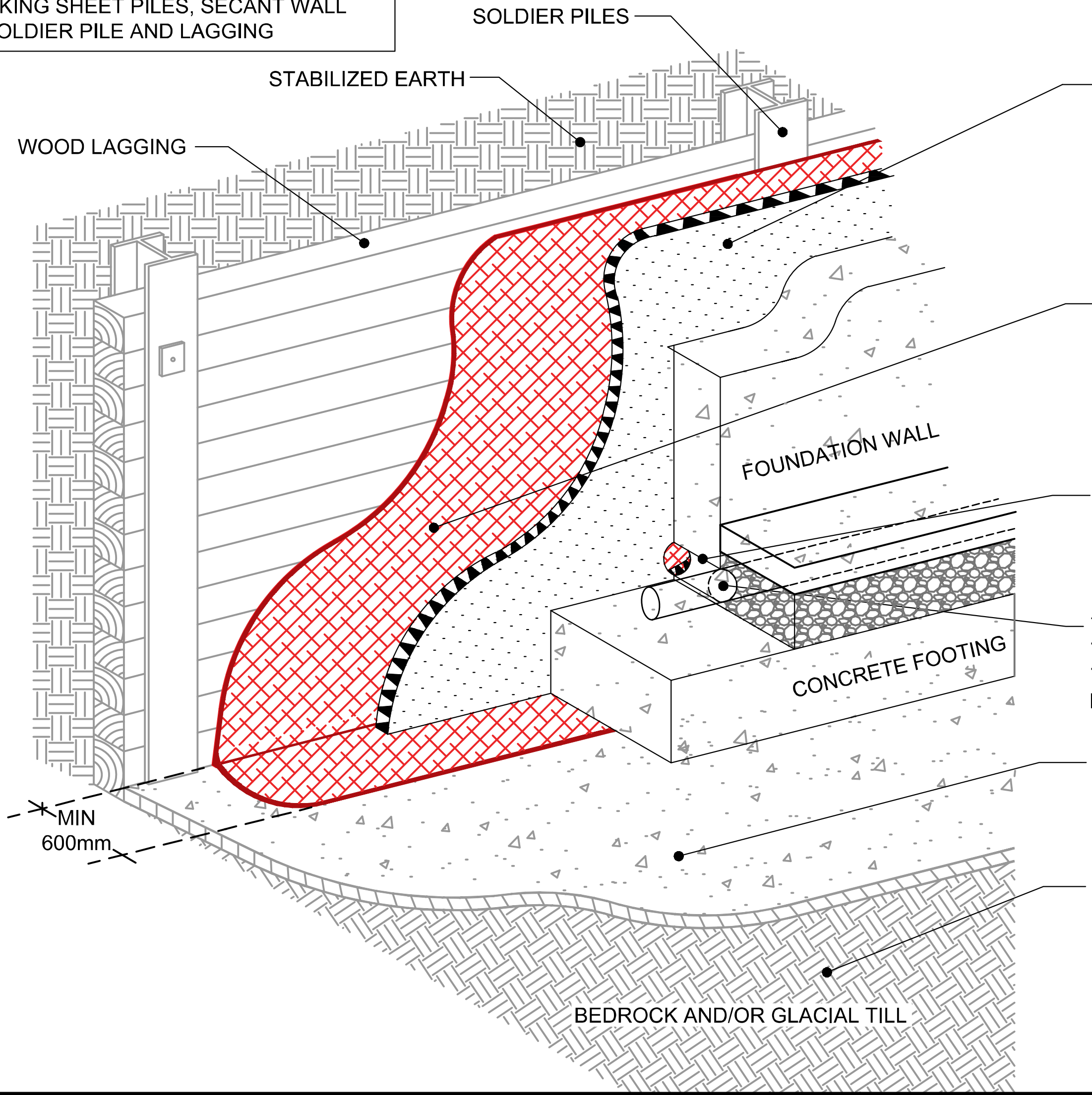


Figure 3 – Shear Wave Profile at Shot Location 34.5 m

TEMPORARY SHORING SYSTEM TO CONSIST OF
INTERLOCKING SHEET PILES, SECANT WALL
AND/OR SOLDIER PILE AND LAGGING



COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH BLUESKIN WP200 AND BLUESKIN PRIMER OR EQUIVALENT.

150miL GRANULAR BENTONITE SURFACING LAMINATED TO 20miL THICK HDPE MEMBRANE. INSTALL IN HORIZONTAL LIFTS TO MANUFACTURERS SPECIFICATIONS IN SHINGLE FASHION WITH THE HDPE SIDE FACING APPLICATOR TO AN ADEQUATELY PREPARED SUBSTRATE SURFACE. WATER PROOFING MEMBRANE TO EXTEND FROM 3.0m BELOW FINISHED GRADE

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

MINIMUM 100mm THICK, 15 MPa CONCRETE MUD SLAB (OPTIONAL)

CLEAN SURFACE SOUNDED BEDROCK, OR GLACIAL TILL TO BE APPROVED BY THE GEOTECHNICAL CONSULTANT

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344
www.patersongroup.ca

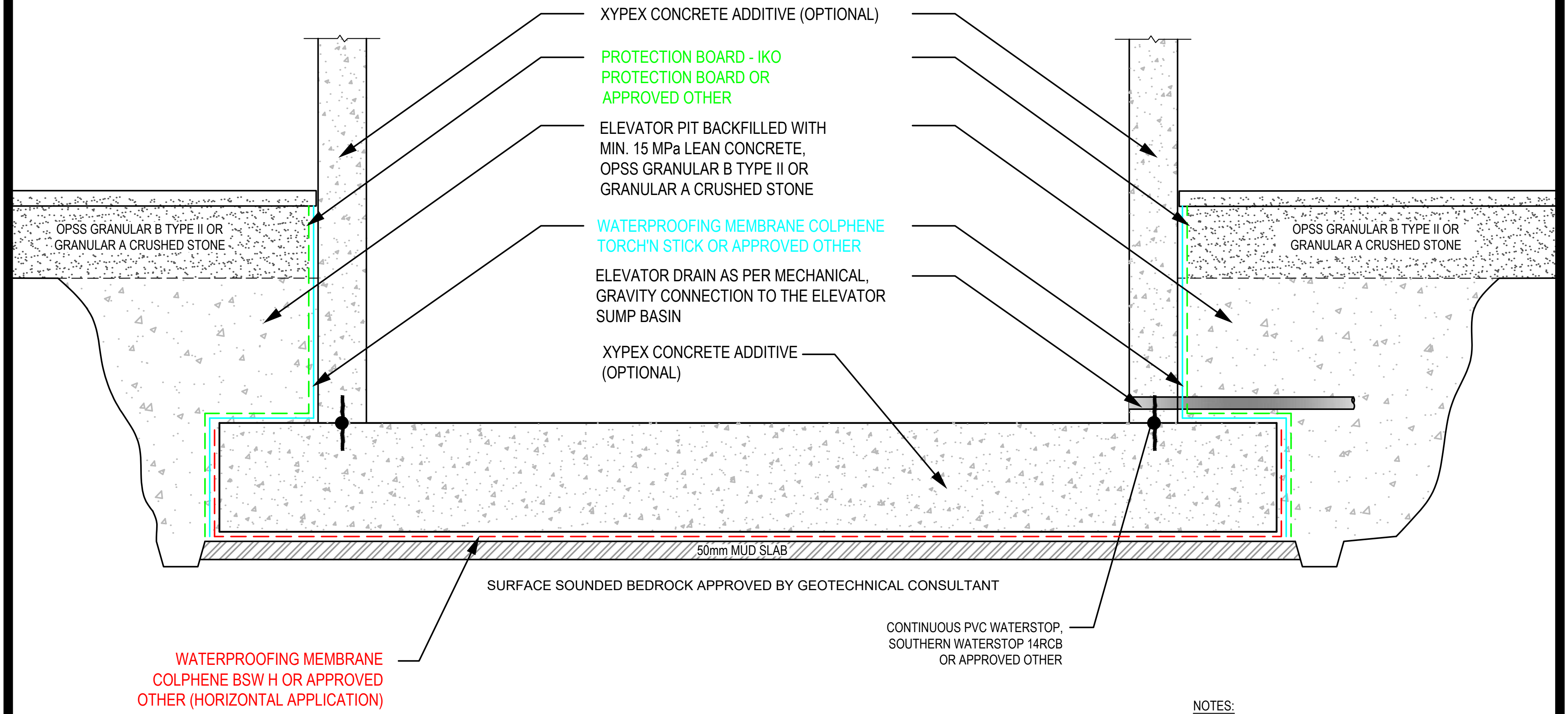
ASHCROFT HOMES
PROPOSED RESIDENTIAL DEVELOPMENT
114 RICHMOND ROAD
OTTAWA, ONTARIO

Title:

GROUNDWATER SUPPRESSION SYSTEM

Scale: N.T.S.
Date: 01/2021
Drawn by: RCG
Report No.: PG2159-1
Checked by: RG
Drawing No.: FIG. 4
Approved by: DG
Revision No.:

p:\autocad drawings\figures\updated foundation drainge template.dwg



NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344
www.patersongroup.ca

TAMARACK (NORMAN) CORPORATION

PROPOSED MULTI-STOREY BUILDING
93 NORMAND STREET
OTTAWA, ONTARIO

Title:

OPTION 1 - ELEVATOR WATERPROOFING DETAIL

Scale: N.T.S.
Date: 01/2021
Drawn by: MPG
Report No.: PG2760-1
Checked by: RG
Drawing No.: FIG. 3
Approved by: RG
Revision No.:

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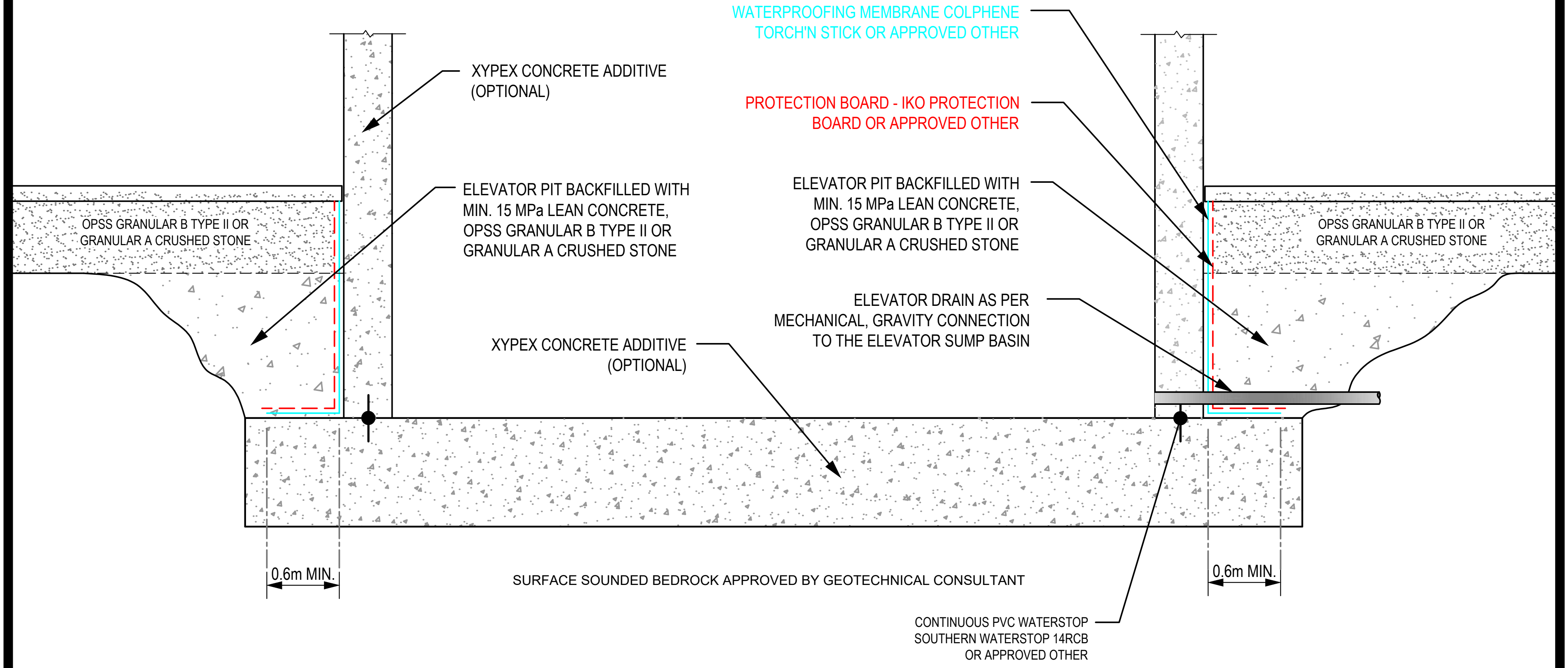

NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344
www.patersongroup.ca

TAMARACK (NORMAN) CORPORATION

PROPOSED MULTI-STOREY BUILDING
93 NORMAND STREET
OTTAWA, ONTARIO

Title:

OPTION 2 - ELEVATOR WATERPROOFING DETAIL

Scale: N.T.S.
Date: 01/2021
Drawn by: MPG
Report No.: PG2760-1
Checked by: RG
Drawing No.: FIG. 4
Approved by: RG
Revision No.:

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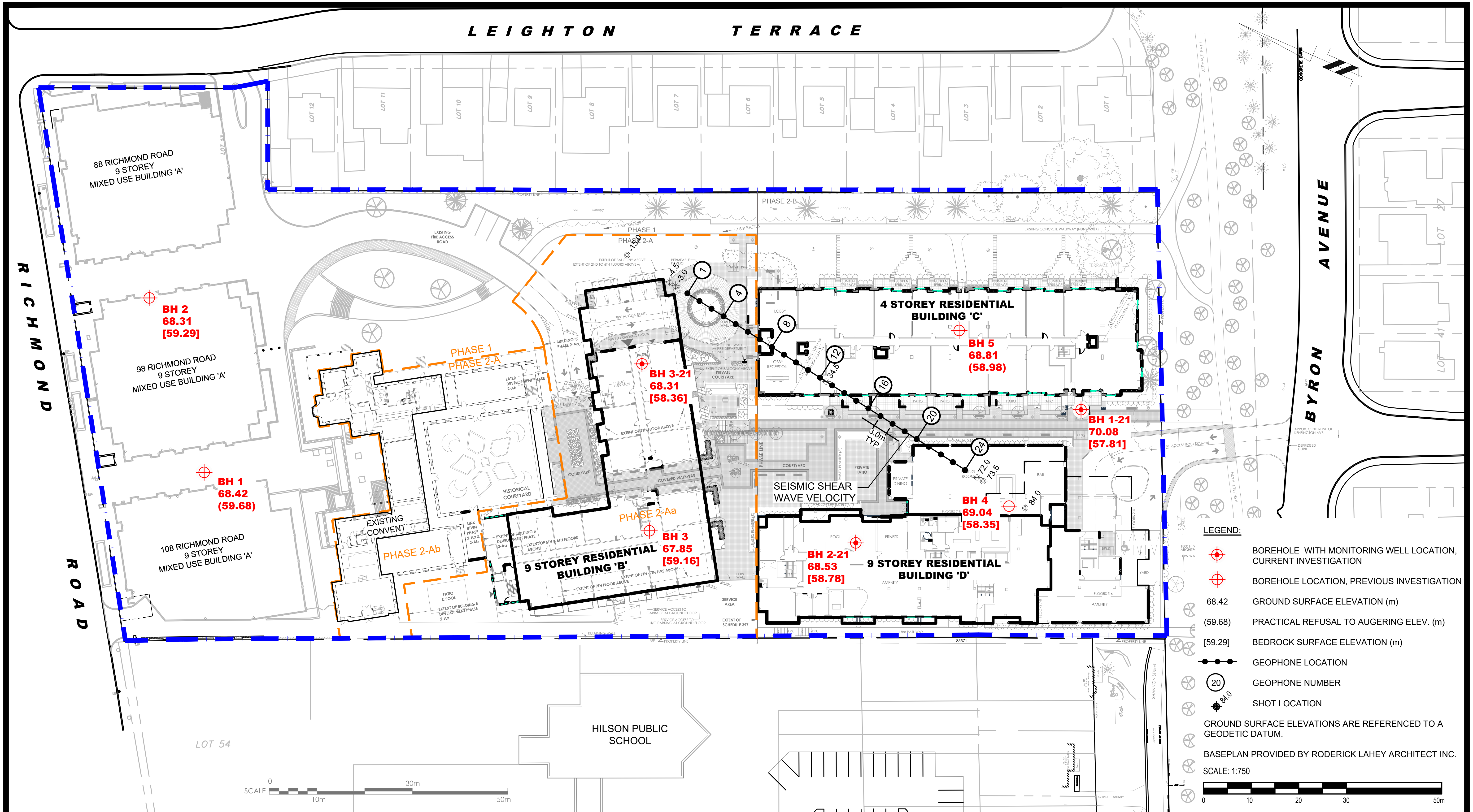

LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION, CURRENT INVESTIGATION
⌖	BOREHOLE LOCATION, PREVIOUS INVESTIGATION
68.42	GROUND SURFACE ELEVATION (m)
(59.68)	PRACTICAL REFUSAL TO AUGERING ELEV. (m)
[59.29]	BEDROCK SURFACE ELEVATION (m)
●–●–●	GEOPHONE LOCATION
(20)	GEOPHONE NUMBER
◆ 84.0	SHOT LOCATION

GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

BASEPLAN PROVIDED BY RODERICK LAHEY ARCHITECT INC.

SCALE: 1:750

0 10 20 30 50m

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
3	BASE PLAN UPDATED	11/04/2022	FA
2	2021 BOREHOLES ADDED TO PLAN	04/02/2021	RG
1	BASE PLAN UPDATED	10/01/2019	RG

ASCHCROFT HOMES
GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT - PHASE 2 - 114 RICHMOND ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATIONPLAN

Scale: 1:750
Date: 07/2010
Drawn by: YA
Report No.: PG2159-1
Checked by: OC
Dwg. No.: PG2159-1
Approved by: FA
Revision No.: 3

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